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Test of a Self-Centering Ring Dome

Architecture

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TEST OF A SELF-CENTERING RING DOME

BY

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B. S. University of Illinois, 1898

THESIS

Submitted in Partial Fulfillment of the Requirements for the

Degree of

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THE GRADUATE SCHOOL

May 21, 1914

I HEREBY RECOMMEND THAT THE THESIS PREPARED BY

Charles Richard Clark

ENTITLED

TEST OF A SELF-CENTERING RING DOME

BE ACCEPTED AS FULFILLING THIS PART ON THE REQUIREMENTS FOR THE

PROFESSIONAL DEGREE OF Master of Architecture

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TEST OF A SELF-CENTERING RING DOME

Introduction

The insistent demand of humanity for a commodity does not often long remain unsupplied. Science usually not only meets the demand; but often anticipating a need, proceeds to create a demand. Occasionally, however, she lags behind due to some obstacle which seems unsurmountable .

When this country wakes up to the enormity of its fire loss (the highest of all of the principal nations), there will doubtless be a demand for a better system of construction. Already in certain quarters there has been an awakening, its demand recognized and fairly well supplied, but in other fields there still remains much to be desired. The mere ability to meet the demand is not always sufficient. It must be so economically met that the application will become general, else the demand will continue. Referring to the before mentioned "better system of construction", which in this case may be interpreted to mean fire-proof construction, we find that in the lighter types of buildings its adoption has not become general, from which we may conclude that the demand has not been met.

It scarcely needs to be mentioned that this system

demands incombustible materials. Of these we have but two in general use, namely, burned clay products and concrete, neither of which is suitable for carrying tensil stresses. We must add therefore other material, usually steel, for taking these stresses, and protect it with incombustible materials; or else resist the force by stresses which are consistent with the material. The writer believes that the chief problem today in fire-proof construction is not the protection of structural members so much as the construction of a light and economical floor of incombustible materials. The solution of this problem will involve either a reduction in the amount of the materials now used, or the use of a new and cheaper material. It would seem that, through the chemist and geologist, we should be so familiar with materials suitable for building operations that relief need not be looked for in the direction of new materials; if so, then we are compelled to adopt the alternative of reducing the amount of those now used. To accomplish this will necessitate a more advantageous use of the materials, for the present working stresses may not be increased with safety.

It is quite generally understood that forces are most economically resisted by tensional stresses, but we have already noted that neither of the usuall fire-proof materials are well adapted to taking tensional stress. They should, therefore, be used in compression if possible, and the system be designed accordingly. The types of construction in which this is feasible are the vault and dome, both of which in their

usual forms produce thrusts at the point of support not conducive to a light form of construction. In the case of the dome, however, this thrust can be taken up by metal rings which, though not fire-proof in themselves, are protected by the structural part of the dome. Here then is a system of construction making logical application of the fire-proof materials and protecting such metal as may be required without addition to the structural parts.

Another feature of fire-proof construction worthy of consideration is that of the forms and centering, which, while adding nothing to the strength or durability of the finished floor, make up a considerable item in the total cost of the work. Their elimination should therefore be considered in any system looking toward cheaper construction.

A study of the foregoing principles has induced the writer to undertake a test of a full-sized ring dome, the results of which are herein set forth.

Historical

Of the early history and development of the dome we know but little. Its introduction is supposed to have been due to the lack of suitable materials for the usual beam and slab construction. Be this as it may, it is certain that nations without a natural supply of timber and stone did most toward its early development.

The Egyptians, among the earliest to develop a permanent architecture, probably knew something of the dome which,

owing to its small elements and the difficulties to be encountered in transporting and handling larger ones, we might naturally have expected them to develop. However, transportation appeared to be the least of their troubles, and a slave driver develops little but a hard heart.

To the Greeks belongs the honor of being the first to develop a truly great architecture. Artistic, highly cultured, and supplied with beautiful marbles in abundance, they chose to develop the post and lintel construction rather than to invent new methods. Their knowledge of the dome was wider than that of the Egyptians, but its use was limited to a few circular temples where beams and slabs would have been inappropriate.

It was quite different with the Assyrians, who, confined between two great rivers where nature had failed to store a natural supply of building materials, were compelled to manufacture the materials with which they constructed. Nature's oversight, however, was made up in other ways. The valley contained not only clay, but also an abundance of mineral oils with which they soon learned to burn xxx brick and tile. With these as their principal building materials, they used the brick dome extensively.

Thus far the use of the dome may have been from necessity, but the beginning of the Roman supremacy marked a new development in architecture, especially in xxx construction. Whatever deficiency there may have been in culture

among the Romans, was more than made up in skill. Imbued with large schemes and thoroughly practical, they constructed buildings which for size and grandeur have rarely if ever been equaled, and to which the architects from the Renaissance to the present day have gone for inspiration. Domes and vaults, which heretofore had been simply coverings,now became a feature of the design, not because of the lack of suitable materials, but on account of their decorative value. The efforts of the Romans in so far as the dome is concerned seem to have been devoted to perfecting the construction and increasing the proportions, which they effectively accomplished. Its adaptation to the square doubtless was never presented to them, else they would have solved the problem. It remained, therefore, for the Byzantines to introduce the pendentive, which effected the transition from the square to the round. While these developments were taking place in Europe a new type of vaulting, the 'timbrel arch' was developed in Assyria and southwestern Asia. However, its use was comparatively limited until its introduction into the United States within the last half century.

Considering briefly the period since the beginning of the Renaissance, we find that the dome has had an important place in the principal architectural monuments. It is also interesting to note that its character has been entirely changed since its first introduction. Then it was a form of construction, now it is seldom more than a feature of the design.

Principles of the Dome.

The term ring dome is often applied to domical structures having metal ribs restrained from spreading by steel shapes forming purlins or rings. It might be applied more properly to the masonry or true dome restrained by hoops, and it is in this sense that it is here used. It is not the intention to discuss here the theory of stresses with the purpose of developing formulae. The information obtained in the test is insufficient either to verify or disprove those formulae proposed by others, and therefore is inadequate for the development of new ones, should they seem to be required. Therefore, without attempting to establish their values, it may be asserted that the following stresses are present:

1. Shear between any segment and the area immediately outside of it.

2. Compression along any element from the center to the base.

3. Compression or tension around any horizontal ring.

The first is taken up by the resistance to shear of the material, and is seldom large enough to endanger the stability of the dome. The compression along an element of the dome is likewise small when uniformly distributed over the area of that element. Its distribution, however, depends upon the relation between the load and the curvature of the element. For example, theoretically an apex load gives a uniform compression

on the elements of a cone. Again, a uniform load on a parabolic arch gives a uniform compressive stress over its section. Likewise a suitable distribution of load, although we are unable to say what the arrangement should be, would produce a similar result in the case of the dome. Even though we knew what the distribution should be, it is not always possible to make such a disposition of the load, and where that is the case the line of resistance of the resultant of compressive stresses will deviate from the center of gravity of the section. As the deviation increases, so will the maximum stress until it reaches infinity just as the resultant falls on the edge of the section.

Experience shows that, under usual loadings, segmental and semi-spherical domes have a tendency to rise at the haunches due to a lowering of the line of resistance at that point. This may be overcome by either of two methods, i.e., by altering the shape of the dome or increasing the load at the haunches. When the first is undesirable, the latter may be resorted to with a corresponding increased load on the substructure. A better method would be to girdle the dome with metal rings capable of resisting, not only the tendency to bulge, but also the third system of stresses, which, if it happens to be tension, is not economically taken by the masonry. These rings, very light in themselves, may be made to produce the same result as an added weight and in addition are capable of varying the stress as required by changing loads such as

wind, snow, etc., provided they are properly secured to the shell. Therefore, the use of rings should tend toward economy over the usual gravity system, not only by a saving in the material of the dome itself, but also in a reduction of the loads and thrusts to be provided for in the substructure.

Application of the Dome to Floor Construction.

A portion, if not all, of the sustaining elements of the usual floor construction are under transverse stress, which, due to its unequal distribution over the section, requires a larger amount of resisting material to support a given load than would be required for a compressive stress such as is found in a dome. A substitution of a dome for flat construction should therefore tend toward economy in so far as the reduction of loads and materials in the floor and substructure is concerned. Besides the above there are, however, at least two other points that influence the amount of material in a floor system, i.e., the span and thickness. The limiting span of the dome is greater than that for any system of transverse floor construction; in fact, as has been already intimated, it was introduced to meet the requirements of great spans. If we consider the thickness of the floor as the vertical distance from the springing of the dome to its crown, it may appear that the dome is inapplicable to floor construction. However, the ratio of rise to span is subject to wide variation, and can be reduced to such an extent that the height of the dome will be little greater for ordinary spans than the thick-

ness of the usual floor construction. In fact, the rise of the center of the dome gives additional height to the room below, so that the actual distance between floors need be no greater than with the beam and slab construction. Likewise, the curved surface is not an objection, but rather an advantage from an esthetic point of view. Also the adaptation of the dome to square and rectangular spaces is believed to be entirely feasible. In fire-proofing qualities the ring dome is fully the equal of slab construction. Constructed as it may be of incombustible materials, the shell fully protects from below the rings on its upper surface, and the filling necessary to level up protects them from above.

Before proceeding with a description of the test, we should note a type of dome construction recently developed in this country. Mr. Guastavino, a native of Barcellona, after a thorough study of the 'timbrel arches and vaults' previously referred to, concludes that, because of their lightness, fire-proofing qualities,and ease of construction, the principle involved in their construction was capable and deserving of a wider application. Due to favorable comment on drawings submitted at the Centennial Exposition and certain other influences, he came to the United states in 1881, and after the usual difficulties incident to such undertakings, succeeded in establishing a satisfactory business as a designer and builder of "cohesive" domes. Only flat vitreous tile 1 inch thick are used in their construction. Two or more layers, depending

on the span, rise and loads, are employed. The tile in the various layers break joints and are laid in a rich mortar on light forms which may be removed almost immediately. This thorough bonding takes up the stresses which tend to distort the dome and prevents any thrust in the supporting members. With vitreous tile and a high grade of cement mortar, all difficulties of construction seem to have been overcome. But it is a little uncertain what the results would be in case settlement developed cracks in the tile, which are in tension in the outer part of the dome.

Purpose of Test

The purposes of the test were as follows:

1. To investigate the adaptability of the ring dome to floor construction, which if practical would insure a light fire-proof floor.

2. To construct the dome without centering.

3. To learn as much as possible about the stresses in a flat ring dome when subjected to loads.

Design of the Test Dome.

Since information on the proposed form of construction is wholly lacking, it was necessary to determine upon details in rather an arbitrary manner. Furthermore the considerable expense incident to, and the time required in, the construction and erection of a dome made it desirable that the type selected for the test should develop as much of the data sought after as possible. Therefore, a rectangular area of 10 ft. x 12 ft., although smaller than many floor panels, was considered large enough to give typical results, and as conforming to prevailing conditions of floor construction.

Increased thickness of a floor, beyond that absolutely required, adds not only to the cost of the floor material but also to that of the walls. It was, therefore, apparent that a ~~floor~~ flat dome was required and accordingly a radius of 50 ft. was chosen. This gives a rise of $3\frac{1}{4}$ inches on the short diameter, $4\frac{1}{2}$ inches on the long diameter and $7\frac{1}{2}$ inches on the diagonal of the rectangle chosen.

The second purpose of the test, namely, to construct the dome without centering, required that the dome be constructed of small units each so formed that it would interlock with and overlap those already placed. The Z shaped tile, as shown in Figure 9, was therefore designed, keeping in mind the early removal of forms and the convenience in

handling. A thickness of $3\frac{1}{4}$ inches was thus decided upon for the shell which, added to the maximum rise, gave $9\frac{3}{4}$ inches as the maximum height of the dome. To this maximum height should be added two inches for cinder fill and 1 inch for finished floor, making a maximum thickness of $12\frac{3}{4}$ inches for a complete floor. It should be noted, however, that, although the floor thickness at the corners is $12\frac{3}{4}$ inches, the thickness at the center is only $5\frac{1}{4}$ inches and therefore the apparent story height would be more than the distance from the floor to the spring line of the dome above.

The tile, about 400 in all, were made of one part of Atlas Portland cement and two parts of local sand, hand mixed, and cast in forms as shown in Figure 9. The forms were lined with tin and the parts arranged so that they could be easily separated when removing the tiles.

The vertical sections of all tiles were similar but their widths varied according to their position in the dome. For convenience in handling, the tiles were made $13\frac{1}{2}$ inches long and not over 8 inches wide. Since the end lap was $4\frac{1}{2}$ inches, each course laid nine inches net. Although designed to break joints with the courses on either side, the tile did not lay up just that way in all parts of the dome, as shown on page 29, which is a diagram of the dome as built.

Grooves $1\frac{1}{4}$ inches wide and deep were cast in the top of the tiles in such a manner that when the tiles were set in place the grooves formed continuous channels in which the rings or reinforcing rods could be thoroughly grouted.

The results of dome analyses which are extant differ so widely as to make it certain that some of the analyses must be seriously in error. An examination of the analyses discloses the possibility that elements have been ignored which may be sufficient to account for so serious a lack of agreement.

Only a little study of the mechanics of the dome shows that an analysis of the stresses would be extremely complicated. It will be apparent that a dome might be made up of (1) independent radial sectors or of (2) independent circumferential zones, the mechanics of the two cases being quite different. In the former case there could be no tension circumferentially across the edge of a sector, but there could be tension radially if sufficient moment were developed to overcome the direct compression. As the angle of a sector becomes small, the action of the element approaches that of an elastic three-hinged arch. As the angle of the sector approaches 360° the radial stresses become less and the circumferential stresses come into importance. In a dome made up of independent circumferential zones, there can be no tension radially across the edge of a zone, but there will be tension circumferentially. If the single circumferential zone, Fig. 1, be considered, it is apparent that circumferential compression will be developed at a, Fig. 2, and circumferential tension will be developed at b in Fig. 2, which represents a radial element taken from the circumferential zone, Fig. 1.

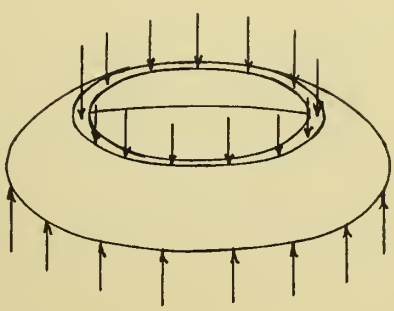


Fig. 1.

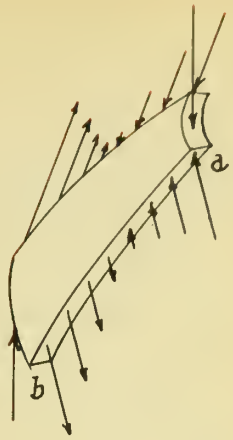


Fig. 2.

It would seem that a dome which is made up in such a way that any tension or any compression ~~xk xkk~~ may be resisted in either a radial or in a circumferential direction will partake of the nature of (1) a dome made up of independent radial sectors and of (2) a dome made up of independent circumferential zones.

A modification of the form of dome made up of circumferential zones is the case in which circumferential reinforcing rods are supplied to take all or a part of the tension which is present in the lower portion of a zone. It is mainly with this modified form of dome that this thesis is concerned.

The foregoing considerations may give some idea of the general nature of the phenomena to be expected in the action of an elastic dome, but an exact analysis involves the solution of a statically indeterminate problem of such complexity that it does not seem feasible to attempt it at the present time. It is believed that empirical formulas derived from even a small number of tests will give results more reliable than any theoretical analysis which has come to the writer's attention. The main value of such tests lies in the measurement of the stresses developed at various places under a given load rather than the determination of the maximum load which may be carried, and the measurement of large deformations is accomplished with less propor-

tional error than the measurement of small deformations. For this reason light construction is desirable in such a test piece, and since no trustworthy formula is available for determining the proper steel area, 3/8 in. round rods placed nine inches on centers were arbitrarily chosen as seeming best to meet the requirements of the test.

Erection

The footings 12 inches wide and the supporting walls 8 inches thick were ~~xxxx~~ constructed of old concrete beams from the Engineering Experiment Station. The notations and cracks visible in some of the photographs are not a part of the record of this test, but are evidences of a previous service rendered. Unskilled labor was used in the erection, and no attempt was made to get better results than would be possible under ordinary working conditions. A rich cement mortar tempered with hydrated lime was used for both walls and dome.

The supporting walls, inclosing a space 10 ft.x 12 ft., were carried up four or five feet high and finished off to correspond to the surface of a sphere whose radius was that of the dome. At each corner and directly on top of the wall the four partial courses #10, were laid, their exact location being determined by the guide shown in Figs. 3 to 6, inclusive. This guide, cut to a radius of 50 ft., was also used for keeping the succeeding courses at the proper elevation. Courses

9 and 8 followed, after which the outer rod, that in course 9, was placed and bedded in cement mortar. Thereafter, the courses were kept one course in advance of the rods and the succeeding courses were placed without difficulty and as rapidly as the tile could be handled. The ends of the rods were anchored by allowing them to lap for 2 to 3 ft., and the laps were staggered in the several courses. After ring #1, Page 29, was completed, a space at the center about 12 inches in diameter remained; this was filled with cement mortar. As soon as completed, the dome appeared to have considerable strength, but was allowed to stand a day or so, after which it was leveled up with 1:3:8 cinder concrete to a depth of 2 inches over the center.

Preparations for Test

In casting the tile, lugs of iron were inserted for strain gage centers, also openings were provided for access to the rods. Two sets of hemispherical centers were used for terminals in taking deflection readings. One was cemented directly to the under surface of the dome, while the other was supported on beams independent of the dome, but resting on the walls. Fig. 10 is a photograph of the under surface of the northwest one-fourth of the dome and shows the condition of the surface and the location of strain gage centers. In addition to the above, plumb lines were supported at the center of the four sides of the panel. These lines extended

to near the ground, where scales were provided for detecting any bulging of the top of the wall.

Loading

Pig iron was used in loading and was applied uniformly in increments of 2000 lbs. per day, until the total reached 12,000 pounds or 100 lbs. per sq. ft. Owing to the necessity of working under the dome, 12 hours at least were allowed to elapse after the application of a load before taking readings. After being loaded to 12,000 lbs. for a few days and showing no signs of failure, the load was increased to 16,000 lbs., or 133 lbs. per sq. ft., which developed the cracks as shown on page 30. This load was entirely removed and a set of readings taken which showed a deflection set of a little less than one-half inch. Next, the north half only was uniformly loaded to 6000 lbs.,or 100 lbs. per sq. ft.,and allowed to stand for a few days.

Readings

The writer wishes to acknowledge here the assistance of Mr. W. A. Slater of the Engineering Experiment Station, whose experience and advice were of very great help in planning xxx and carrying out the test. All readings were taken by him, and it is therefore believed because of his extended experience that the results obtained are a reliable record of the behavior of the dome.

The locations of the various readings are shown on page 29, the top of the page being the north end of the dome.

As before stated, the loads were applied in increments of 2000 lbs. uniformly distributed. For each increment, up to and including 12,000 lbs., readings were taken at the points indicated (something over 100 in all), using a standard strain gage with a 4-inch gage length. The usual precaution for avoiding errors due to temperature, etc., i.e., checking with standard invar bar at short intervals, was observed.

The corrected readings on the concrete are plotted on pages 31 and 32. Readings taken parallel to the elements of the dome, or radially, are shown on page 31. Readings taken in a circumferential direction are shown on page 32. Readings across joints, both radially and circumferentially, are shown on page 33. The radial and circumferential - load deformation - curves are plotted directly from the readings for the point indicated by the number just below, and indicate the stress in units of 100 lbs. on the assumption that the modulus of elasticity for the concrete = 3,000,000 lbs. To get the unit deformation, apply the scale as noted on each page. The curve at the bottom is plotted with the courses as abscissa and the average unit deformations as ordinates. At the extreme left of this curve is a scale showing the stresses in hundreds of pounds per sq. in., assuming E = 3,000,000 as before. The curves are complete for all loads up to and including 12,000 lbs. Readings for the maximum load of 16,000 lbs. were taken on only a few of the outer rods, the data for points under the dome not being considered worth the risk.

Deformation curves for all points on the steel rings are shown on page 34, and at the bottom also is shown the curves for the average deformations for the various loads. Likewise on this page, the stresses developed in the steel for the various loads are shown at the extreme left of the lower curve, in thousands of pounds. The values are based on the assumption that the modulus of elasticity for the steel = 30,000,000 lbs.

No deflection curve is shown, but readings were taken at the ten points A to J, inclusive, for each load and varied almost directly as the load up to 13,000 lbs., the maximum for that load being 0.13 inch. For the 16,000 lb. load it reached 0.7 inch, but regained 0.32 inch after the load was removed.

No cracks were visible until the load reached 16,000 lbs., and those shown on page 30 were the result of this load. Although the dome was beginning to fail as indicated by the cracks, it was decided to learn as much as possible about its stiffness, and to this end the north half only was loaded with 6,000 lbs., or 100 lbs. per sq. ft. As no particular change was noted in the readings, it would appear that although the dome was somewhat weakened by the maximum load, it still possessed sufficient strength to resist unsymmetrical loads. After each increment of load, the plumb-bobs at the centers of the walls were carefully observed, but no bulging could be detected.

A number of readings, not shown on the diagram, were taken to see if the ends of rods slipped in the concrete. These seem to indicate that where properly bedded in the concrete and where the lap was at least 2 ft., there was little or no tendency to slip. In two or more cases where there was very little lap and where the ends were bared for the purpose of taking readings, the slip was sufficient to have a detrimental effect on the dome.

Deformations and Stresses.

In the discussion of stresses, a value of 30,000,000 lb. per sq. in has been assumed as being nearly correct for the modulus of elasticity of steel. For the concrete 3,000,000 lb. per sq. in. has been assumed rather arbitrarily. The only value in reducing the deformations to stresses lies in the fact that most engineers think more easily in terms of stress than in terms of deformation.

The unit deformations in a radial direction were generally small. They were largest in the outer rings where compression occurred. The curve at the bottom of page 31 shows a distinct tendency for the deformation to decrease with the decrease in distance from the center of the dome, changing from compression to tension at a position 2 1/2 to 3 ft. from the center. It changes back to compression at a point still closer to the center, and in this respect the results do not follow any known law. The deformations across the joints in the radial direction are not more marked than are the deformations measured xxxxxxxx within a single tile. This would indicate a satisfactory mortar joint between concentric rings.

The curves on page 32 show the circumferential deforma-

tions in the concrete to be very small and very erratic in their distribution. Some explanation for the smallness of the deformations and for the erratic distribution will be found in the fact that in most cases the opening between tile (as shown in the lower set of curves, page 33) was quite marked; also in the fact that generally speaking the cracks (page 30) took a radial direction. The maximum circumferential stresses in the concrete at 12,000 lb. load, as shown by the lower curve of page 31, were 150 and 120 lb. per sq. in. in compression and tension, respectively.

The stress indications for the steel rings are more definite than those for the concrete. The curve at the bottom of page 34 for the 12,000 lb. load indicates a comparatively uniform variation of stress from a value of 8000 lb. per sq. in. tension at a point 7 ft. from the center to 5000 lb. per sq. in in compression at a point about 1 ft. from the center of the dome.

The position of gage lines 90, 94, 96 and 100 is such that stresses at these places should not be considered in this comparison. With these stresses disregarded, the curves at the bottom of page 34 would be more regular than they are. An inspection of the load stress curves for these gage lines shows that their behavior was quite different from that of other gage lines in ring No. 9.

It is believed that in the curves showing stress distribution, it is possible to distinguish; (1) flexural action, in the decrease in the radial compression with increasing distance from the outside ring of the dome; (2) arch action, in the fact that with little fixity of the structure at the support there was considerable radial compression in the outer rings, and;

(3) circumferential stresses, which would appear to be a characteristic of the action of an element of a ring dome.

Conclusions

From a single test, it is hardly possible to draw reliable conclusions, but from the information obtained, it would seem that the following indications may be recorded:

1. That the stresses along the radial elements of the dome are compressive, and that they increase toward the outer edge.

2. That the circumferential stresses taken by the steel in tension are largest near the outer edge; that near the center the stresses are in compression; and that somewhere between these limits is a neutral zone.

3. That circumferential stresses in the masonry are small, and that their behavior is indeterminable from the data at hand.

The test further shows that it is entirely practicable to build such a structure without centering. Less than 1% of the tile were broken in laying and no difficulty was experienced in placing them. The material may be said to have been somewhat unfavorable, since the cement tile were heavy and quite easily broken by a sharp blow. Semi-porous clay tiles would be lighter, stiffer and more easily placed, owing to the capillarity of such material. These tile could be produced in the usual brick or tile machine, since the vertical section along an element of the dome is uniform for all courses. The width of the tile and the direction of their sides vary for the several courses, but the courses are similar for each dome. The cut-off machine would readily take care of such variations. The only unusual feature of the tile as formed for the test is the curvature of ends, i.e., they conformed to the true curvature of the course in which they were used. Whether this feature is essential or even desirable, has not been determined, but at least two methods are available for producing it. One, a variation in speed of the sides of the plunger if that type of machine were used. The other a variation in thickness of the ends of the die where the auger machine is used. While the tiles for the test were of varying widths and made especially for each course, it has not been established that this is necessary. It would seem advisable to break joints as much as possible, but otherwise the tile might be interchanged.

A change which it is believed would be advantageous is the substitution of several laps of wire for each of the rods. This would have two advantages; first, it could be placed without the use of tools for bending; and second, only one strand of the ring would be spliced, all others being continuous.

As regards the adaptability of the ring dome to floor construction, it is, of course, realized that one trial does not establish a principle applicable to all sorts of varying conditions or circumstances. But, without experience or the best of materials, to have supported a load of 12,000 lbs,, (equal to 100 lbs. per sq. ft.) on a construction 2 1/4 inches thick, with less than a pound of steel per sq. ft., which was considerably more than required, does not presage a fundamental defect. The maximum deflection for this load was 0.13 inches or just 2/5 the usual allowable deflection. The stresses in the steel and concrete for the load of 100 lbs. per sq. ft. were far below the allowable. When the load was increased to 16,000 lbs. or 133 lbs. per sq. ft., cracks appeared, and a maximum deflection of 0.7 inch was recorded. These signs of failure, however, were due quite as much to slipping of the ends of poorly anchored rods as to the load itself. With such improvements as are usual in the development of new forms of construction, it is believed that the dome would prove to be a light, economical and fire-proof floor system for many classes of structures.

The writer wishes to state with regard to the dome built and tested that the following are original with himself:

(1) The general scheme, (2) the idea of a floor easily built without forms, (3) the conception and detailed design of segmental blocks for this purpose, and (4) the reinforcing with circumferential rods. The test was made on his own initiative and at his own expense.

Fig. 3.

Fig. 4

Fig. 5

Fig 6

Views During Erection.

Fig. 7 View of top of dome ready for cinder fill.

Fig. 8 Dome with 16000# live load,

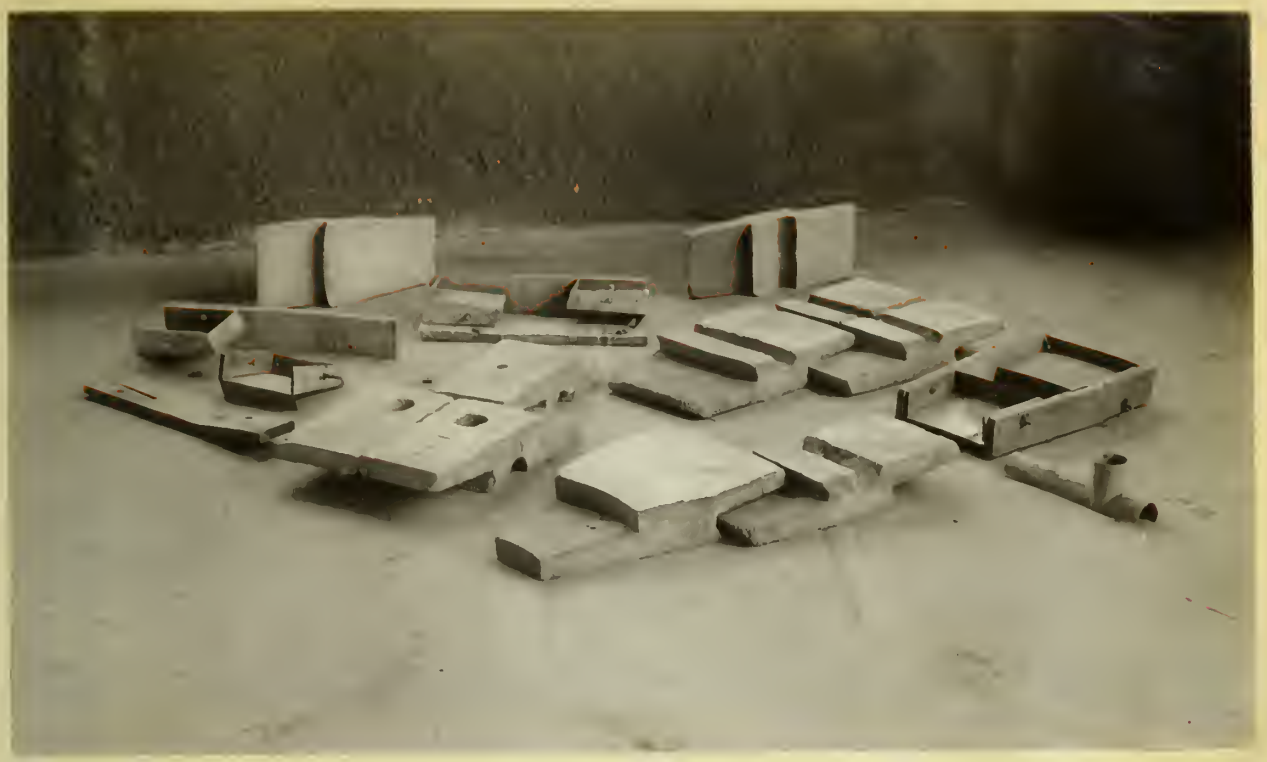


Fig. 9 Views of tiles and forms.

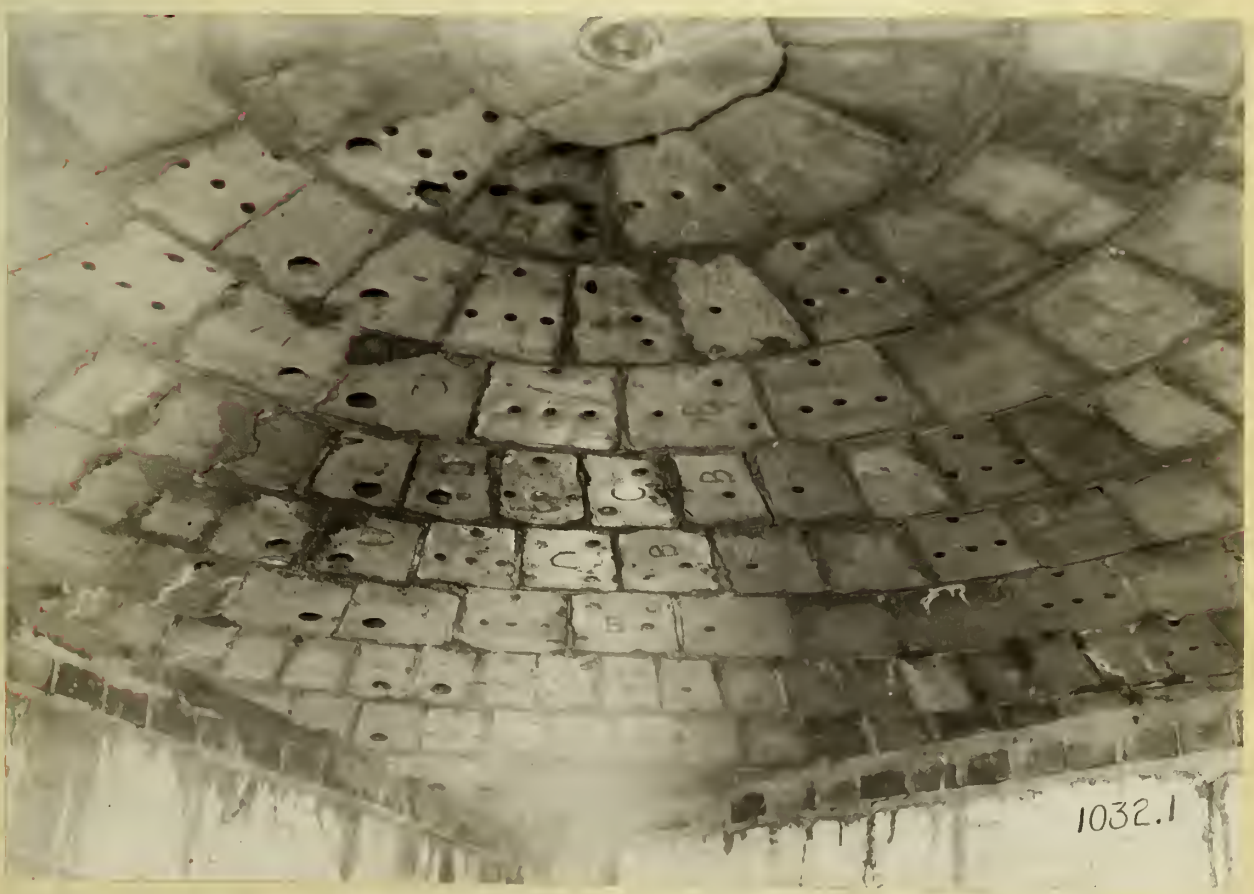
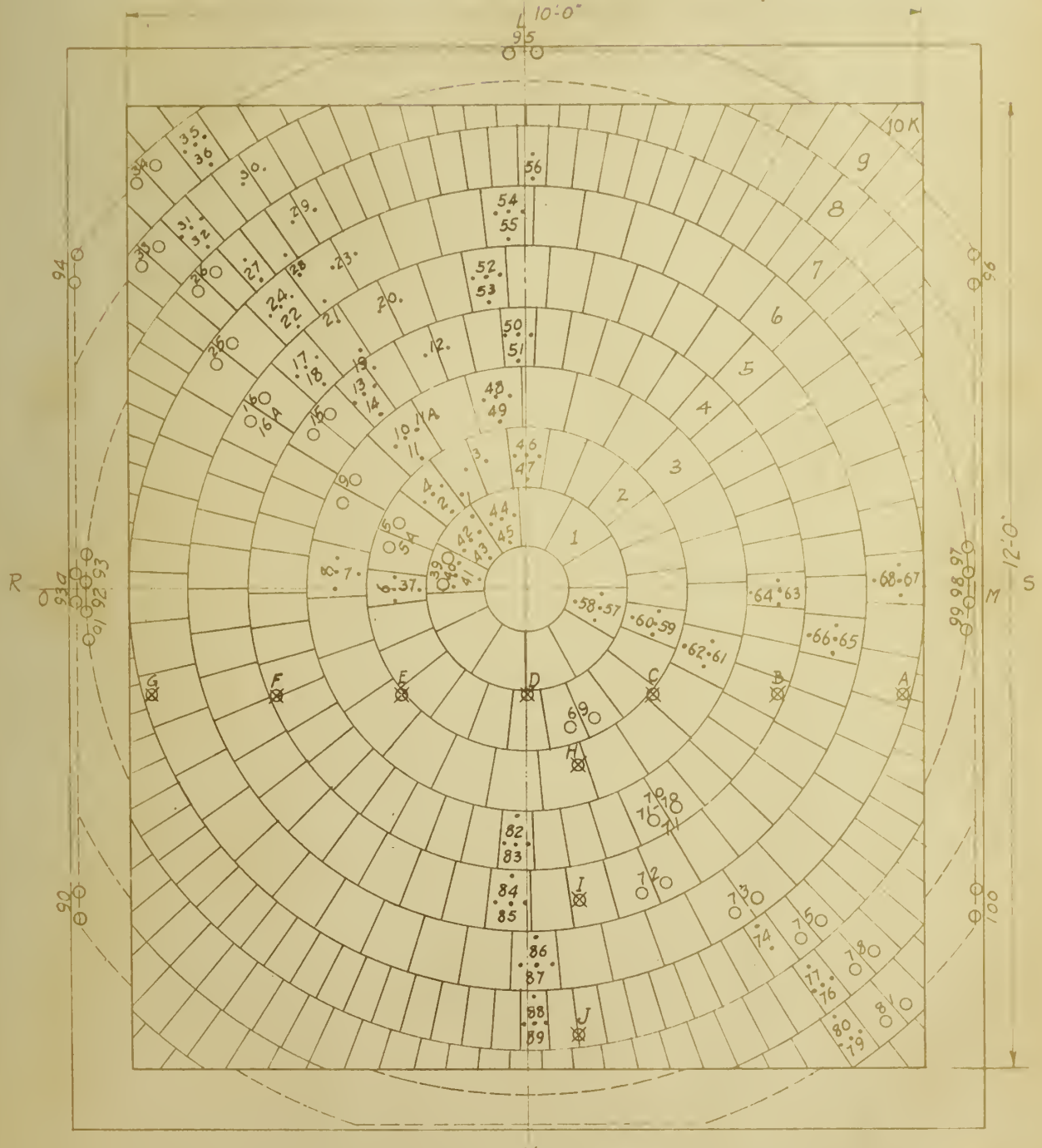
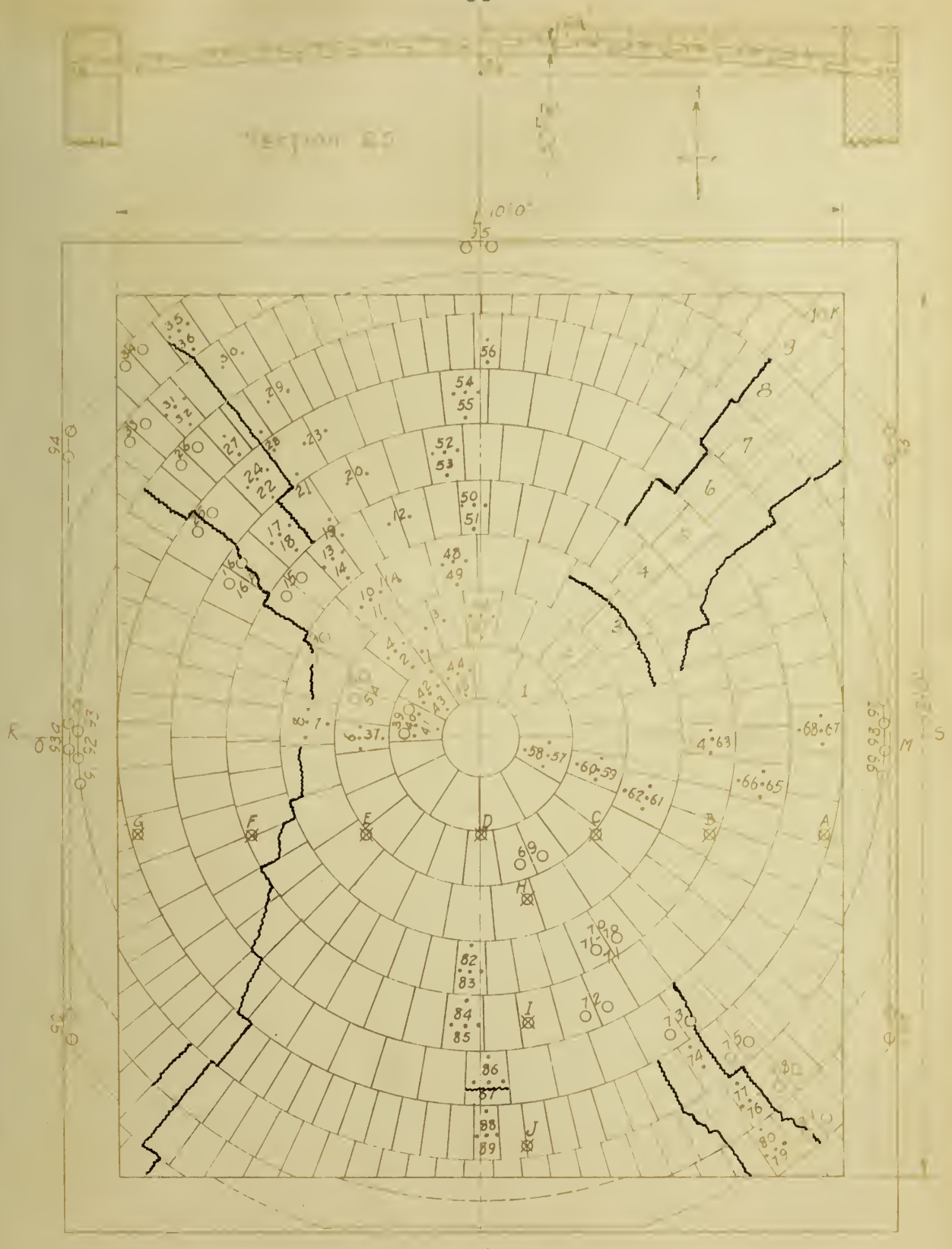


Fig. 10 Under surface of dome showing strain gage centers.



• Indicates strain gage centers in concrete.
○ " " " " on steel rods.
⊠ " location of deflection readings.

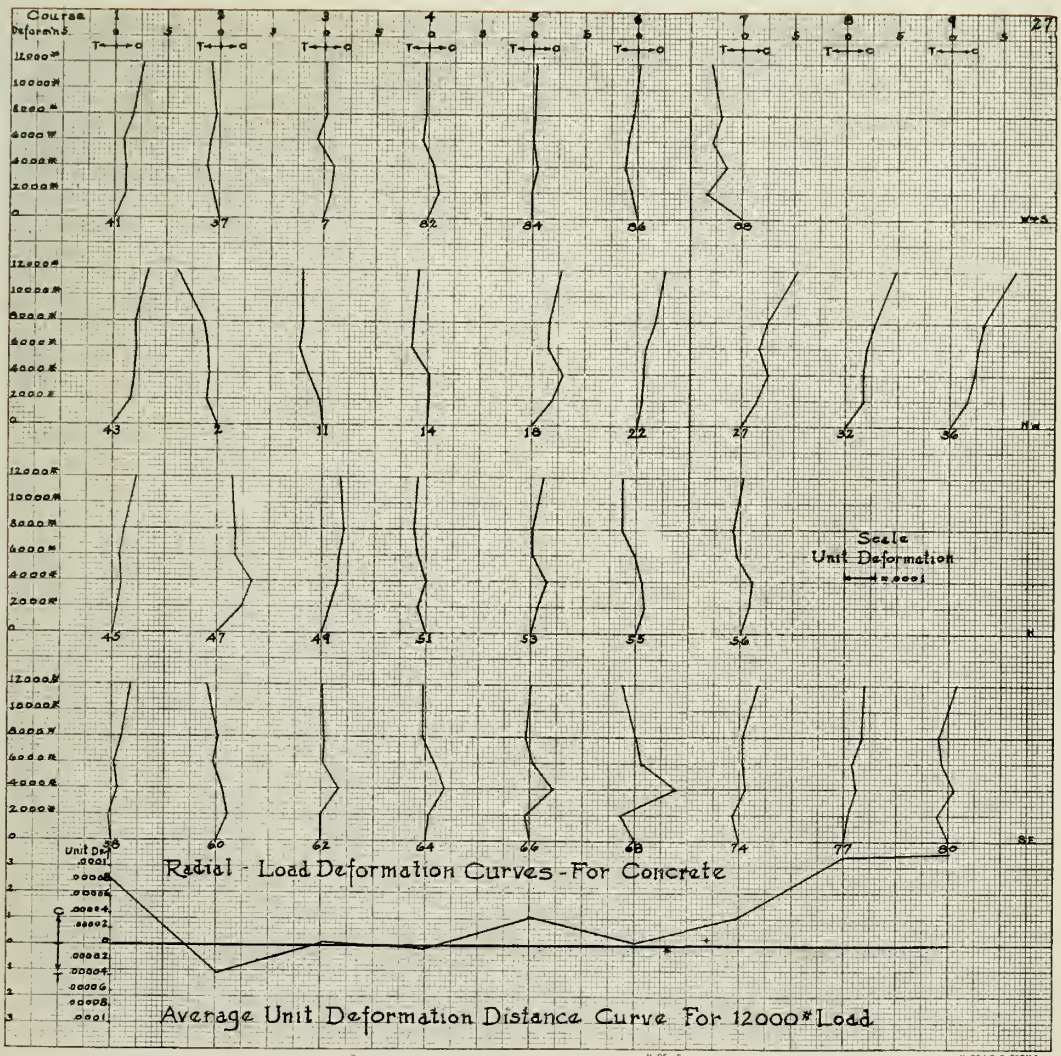
PROJECTED PLAN OF CEILING OF SELF-CENTERING RING DOME
SCALE - $\frac{1}{2}$" = 1 FT.

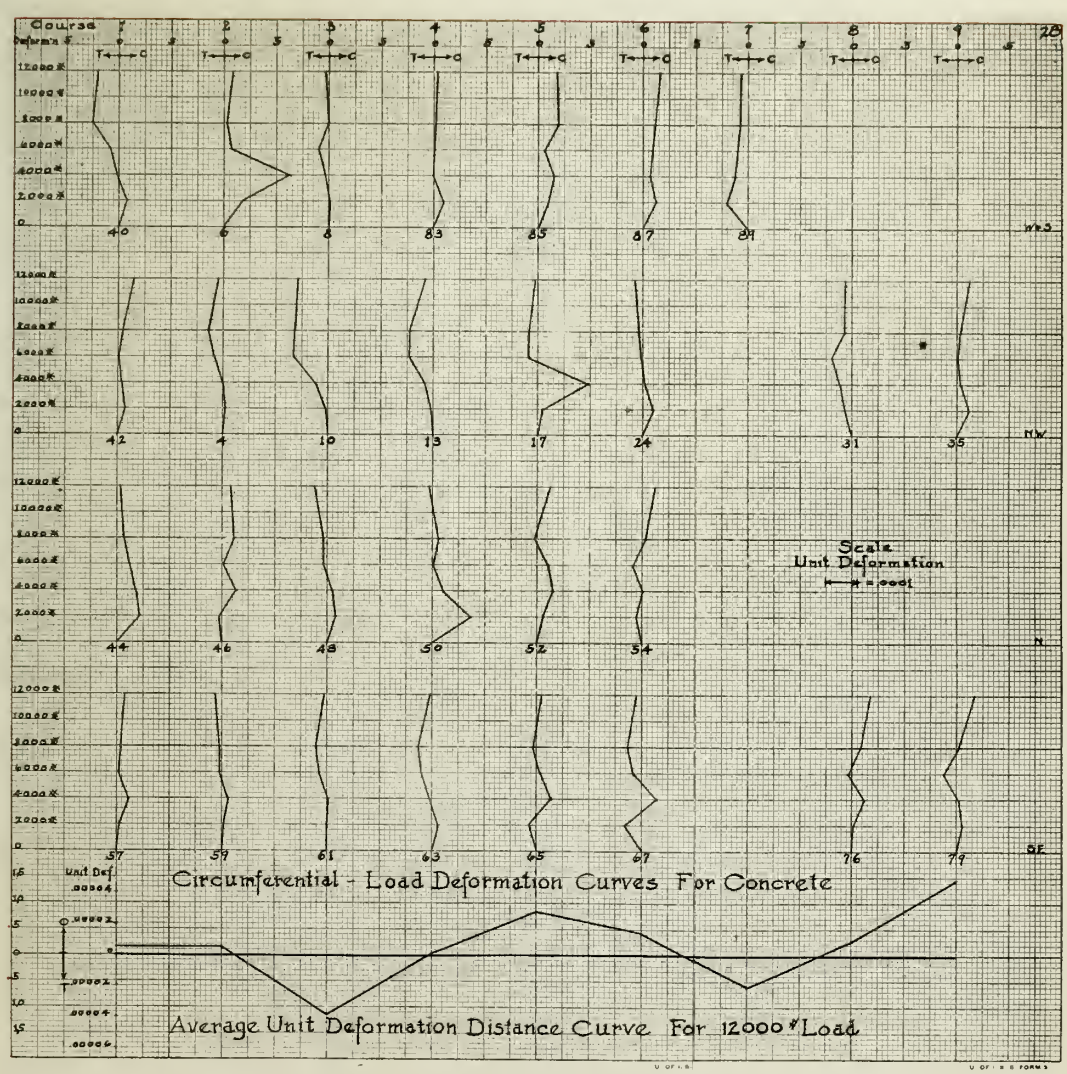


PROJECTED PLAN OF CEILING OF SELF-CENTERING RING DOME

SCALE - 1/2" = 1 FT.

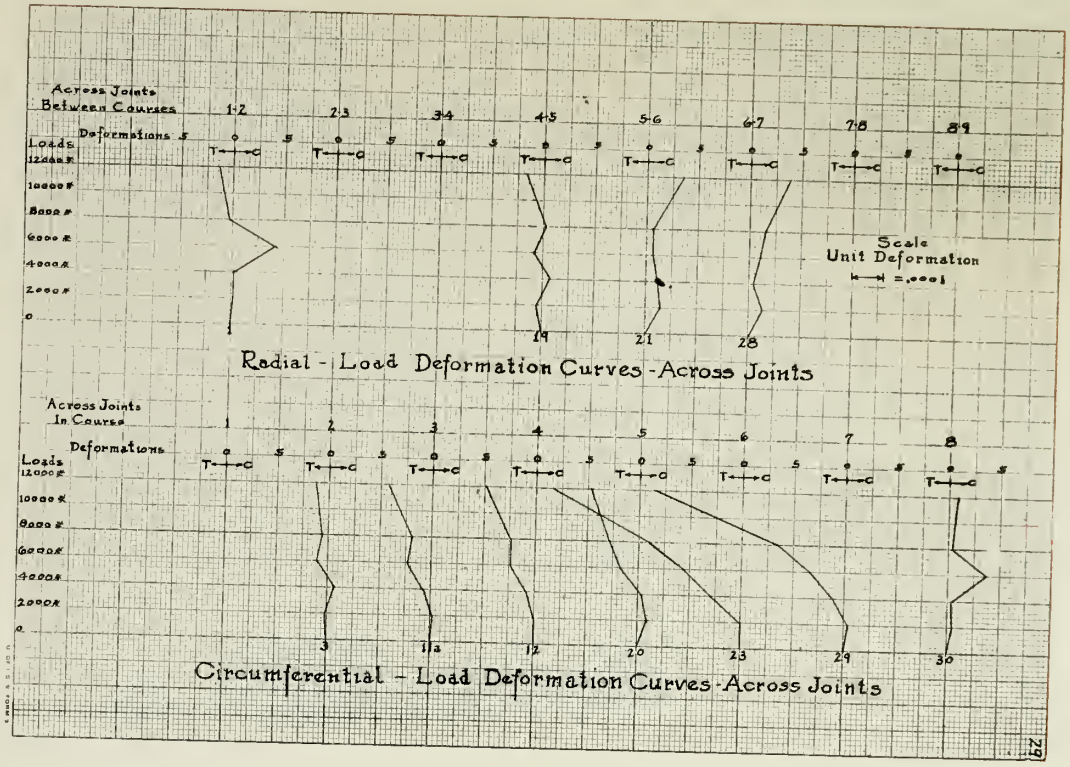
CRACKS FOUND AFTER APPLICATION OF 16000* LOAD

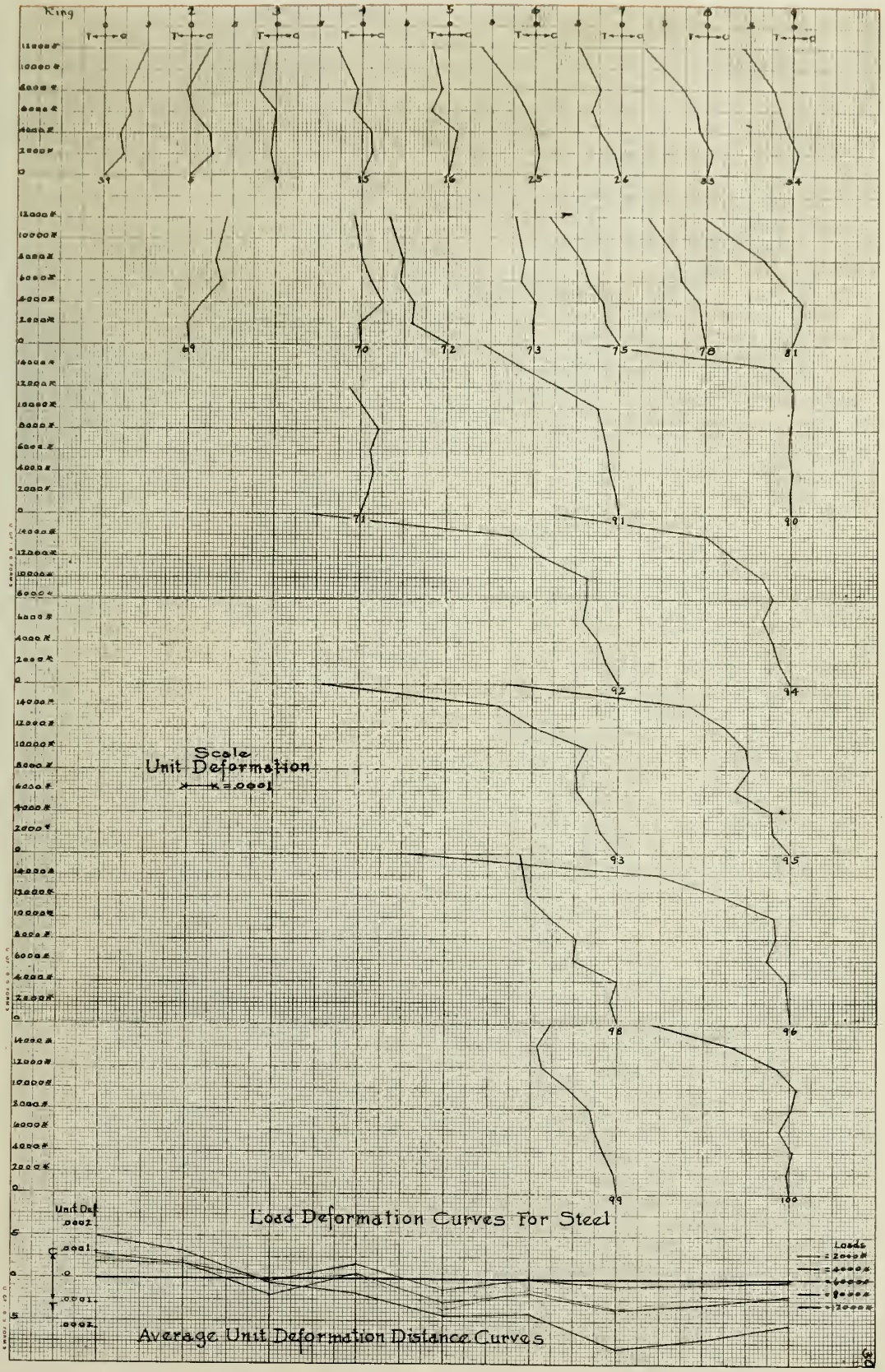

Course
Deformn's
Scale
Unit Deformation
Radial - Load Deformation Curves - For Concrete
Average Unit Deformation Distance Curve For 12000# Load



Circumferential - Load Deformation Curves For Concrete

Average Unit Deformation Distance Curve For 12000# Load


Across Joints
Between Courses
1-2
2-3
3-4
4-5
5-6
6-7
7-8
8-9
Loads
Deformations
12000 #
10000 #
8000 #
6000 #
4000 #
2000 #
0
T
C
Scale
Unit Deformation
=.0001
19
21
28
Radial - Load Deformation Curves - Across Joints
Across Joints
In Course
1
2
3
4
5
6
7
8
3
11a
12
20
23
29
30
Circumferential - Load Deformation Curves - Across Joints


Ring
Scale
Unit Deformation
.0001
Load Deformation Curves For Steel
Average Unit Deformation Distance Curves

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